WATER SURFACE PROFILES FOR LAND DRAINAGE CHANNELS

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WATER SURFACE PROFILES FOR LAND DRAINAGE CHANNELS

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Site Conditions Requiring Special Water Surface Profiles

Frequently, the design of land drainage channels requires knowledge of water surface profiles at particular sites to determine their effect on side drains. Such knowledge permits suitable adjustments to be made in the design of the drainage channels. Typical sites involved are:

1. Natural channels and floodways.
2. Outlets into natural channels, floodways, lakes, tidal waters or artificial impoundments, such as reservoirs, mill dams, etc.
3. Partially or intermittently improved sections of natural channel, such as cutoffs on a meandering stream.
4. Natural or artificial barriers or constrictions, such as cuts through rock ledges, culverts, bridges, raised roadways, levees and dikes, floodgates, weirs, etc.

Methods of Computing Water Surface Profiles

There are several established methods for computing water surface profiles. Reference should be made to SCS-National Engineering Handbook, Chapters 4 and 5 in Hydrology and Hydraulics, Kings Handbook, (Fourth Edition), Chapters 7 and 8, and other hydraulic texts.

A simplified version of the Step Method, modified to account for velocity head, provides an accurate and suitable method for determination of water surfaces in drainage design. This procedure is covered herein, along with the solution of a typical problem.

Basic Procedure in Modified Step Method

As in other methods, no exact solution of the water surface profile can be made. However, by proper care in selecting and measuring the hydraulic factors during a survey, the computed stages may be determined within practical limits. In general, procedures involved are:

1. Divide the length of channel or channel and floodway into a series of consecutive reaches so that the reaches are nearly uniform in hydraulic elements such as size, shape and surface roughness, and have similar flow conditions which may be steady or unsteady, uniform or turbulent. Thus, bridges, weirs, etc. are constrictions which are set up separately with head and tailwater sections as reach limits.
2. Determine the water stage at the control point for the discharge under consideration. This stage is determined at some critical section, such as a structure, or is computed for a measured reach of channel or floodway. The length of

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channel reach under consideration may need to be extended or reaches added so as to include the location of a control point.

3. Determine the water stages of the upstream and downstream ends of an initial reach, with the control point as the water stage at one end of the reach. In nearly all land drainage systems subcritical or retarded flow exists. For this type of water surface profile, referred to as a back water curve, the control point will be the downstream stage of the reach. The difference in stage of the water between reach ends will be equal to the computed potential energy head loss through the reach. This head loss, when added to the control point elevation, will give the upstream water stage of the reach.

4. Compute water stages of upstream and downstream ends of successive reaches. Since the computed upstream stage of the initial reach becomes the downstream stage of the next reach, successive reaches can be computed in the same manner as was done for the initial reach, for as many reaches as are necessary.

Kinds of Flow Conditions for Determining Head Losses

In order that proper head losses may be determined, flow in each reach must be classed as to types. Flow may be Steady or Unsteady. In Steady flow, discharge at a given time for successive cross sections is constant, or the quantity entering and discharging from a reach is the same. Based on the principle of Continuity of Flow, $Q = A_1V_1 = A_2V_2 = A_3V_3$, where 1, 2 and 3 are successive reaches. Determination of the water surface profile is dependent on steady flow conditions, and reaches must be so established that the Q can be considered as constant throughout.

Discharge affected by tidal streams or discharge from a floodwater retarding structure is an unsteady state of flow. However, sufficiently accurate solutions can be established by considering the total flow during an interval of time as a constant mean flow.

Velocity in the Continuity Equation is taken as the mean velocity and is assumed to be the same for every particle of water moving through a cross section. Actual velocities of all particles vary and do not necessarily move through the cross section in the same direction, so that mean velocity in itself does not reflect the total loss in energy. The greater the turbulence, the more diverse the velocities and flow paths.

When the area and shape of successive cross sections remains constant, water particles tend to move in parallel lines with negligible turbulence. Energy losses are then constant as long as friction losses along the faces over which water moves remain constant. Flow is then considered to be uniform and is the condition generally assumed in regular channel design.

When area or shape of successive cross sections varies, flow is nonuniform. However, gradual changes in cross section, such as caused by slight bank scour at bridges or the slight deepening, widening and narrowing of natural channels, result in only minor differences in velocity through succeeding cross sections. Such flow is gradual nonuniform. By limiting reaches to short lengths, flow can be considered as uniform for each reach. Usually, water slopes under 1 percent and velocity differences between reaches of less than 20 percent result in turbulence losses insufficient for consideration other than in the regular friction factor. Greater error may occur in judging the friction factor "n" than in other losses.

When the area or shape of successive cross sections changes abruptly, such as flow through a narrow bridge, velocity changes are sudden, causing turbulent nonuniform flow. Losses in such reaches are generally included with friction losses covering a particular type structure, such as for special cases of orifice flow, weirs and transition sections.

Determination of Head Loss Within Reaches

Flowing water possesses energy by reason of pressure which it exerts through velocity, weight of its confined mass, and the position above a datum. As shown in Figure 1, the Potential energy gradient conforms to the hydraulic gradient of the water surface, and the Kinetic energy gradient for subcritical flow will be an addition to and will lie above the water gradient. Thus, the Manning formula may be used to establish the water gradient and total potential energy head loss through reaches with uniform flow where only friction loss must be accounted for. This loss of head within a reach = $Ls = \frac{Ln^2v^2}{2.2082r^{4/3}}$. The additional energy head must be accounted for at constricted reaches. This can be done as a velocity of approach head allowance in the orifice, weir or other flow formulas covering head losses through such constrictions. Since water surface profile calculations for retarded flow progress upstream, an estimated approach velocity, which is the velocity of the next upstream reach, must be used and this corrected by a second trial, if necessary.

In case "n" values or relationship of wetted perimeter to cross section area of different parts of the total cross section differ substantially, the section should be divided in accordance with such differences and the total of the individually computed discharges should equal the required discharge of the whole section. A typical example of such a cross section is shown in Figure 2.

In computing the water slope, the cross section at the lower end of the reach may be used as applying to the whole reach, since small changes in area and hydraulic radius of short reaches on flat slopes permit considerable adjustment in the velocity without affecting accuracy. Thus with Q and A given, V can be computed without retrial, as required in the usual step method. However, when several

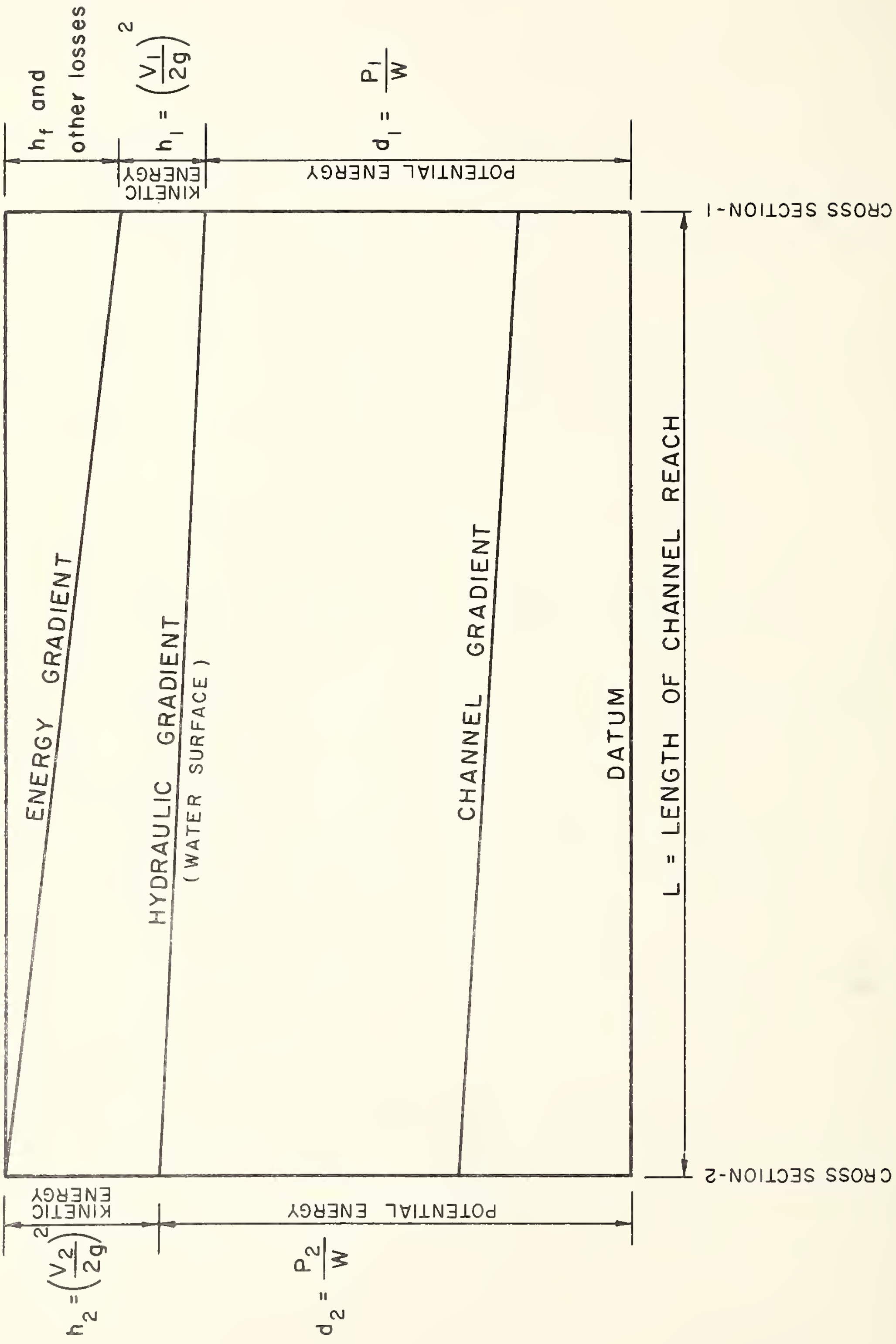


FIGURE 1 — ENERGY OF OPEN CHANNEL FLOW

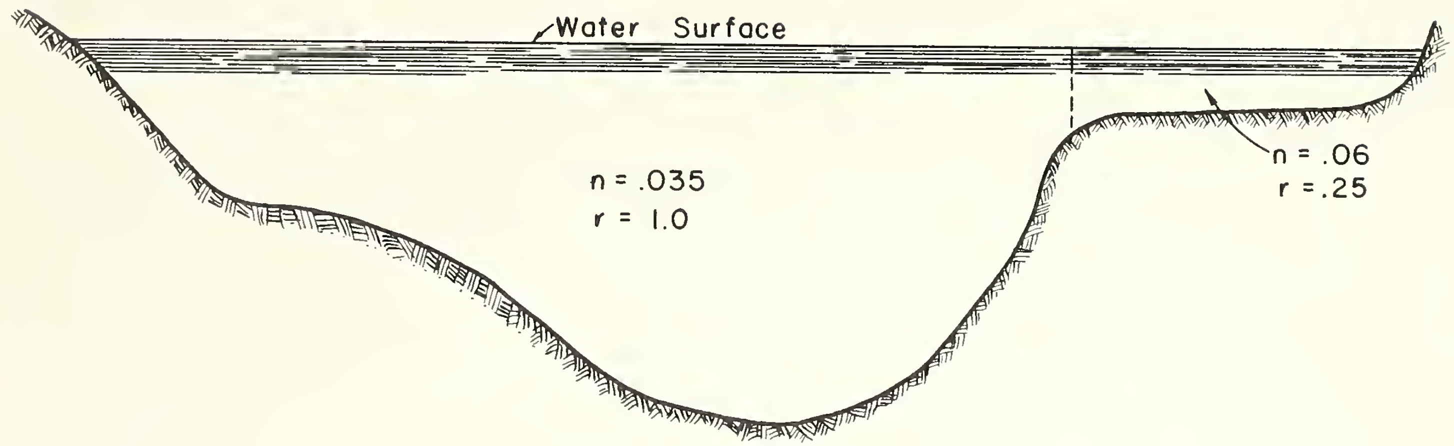


FIGURE 2 – CROSS SECTION OF STREAM CHANNEL AND FLOOD PLAIN

Q values must be calculated for several parts of a cross section, an estimated slope should be selected and the corresponding velocity of each part computed. If the total Q value does not equal the required Q, another trial must be made. The length of reach "L" should be taken as the distance along the course of principal flow, which is usually the channel in an overflow section. However, when the actual flow paths of the flood plain are different they will have different slopes between the top and bottom of the reach. The difference in water elevation between the top and bottom of the reach should be determined from the channel slope and the slope of flood plain flow recomputed from this difference.

Establishing Water Stage by Trial Water Surface Profiles.

Frequently there is no gage or other recorded elevations of flow for use as a control point to start computation of the water surface profile. The control point can be established with reasonable accuracy by computing several water surface profiles for a series of reaches over a measured section of channel and floodway at the lower limits of the desired profile. Usually three or more trial runs are carried through a number of reaches with the assumed starting stage changed by, say, 1/2 foot intervals. With all hydraulic elements essentially the same except for cross section areas and corresponding velocity, the computed slopes of successive reaches will eventually converge upstream in a common elevation which is on the water profile.

It should be noted that the greatest error in establishing such a stage, or in computing the whole water profile, comes in the evaluation of the roughness factor "n", which requires the greatest exercise of judgment. Therefore this value should be determined with care. Whenever possible, profiles of available flood marks should be obtained and used as checks on estimated "n" values.

Solution of Typical Problem

The following is a step-by-step solution of a typical problem with both channel and overbank flow and varying roughness factors "n" considered in each reach.

Situation.

Forty-two acres of partially developed truck cropland require protection from overflow and inadequate drainage. Protection is to be provided by means of a levee and pump drainage. The tract lies on the left bank of a low-gradient stream immediately upstream from a bridge and elevated road (See Figure 3). The levee will be located at the streambank, extending from the roadway to a wooded tract upstream and thence to high ground. No overtopping of the road has been known to occur since construction of the road and bridge 60 years previously. The only available stream discharge records are from a U.S.G.S. gage on the bridge approximately 2 miles upstream (See Table 1).

Problem.

Determine the water surface elevation along the course of stream adjacent to proposed levee for 25-year discharge frequency in order to establish:

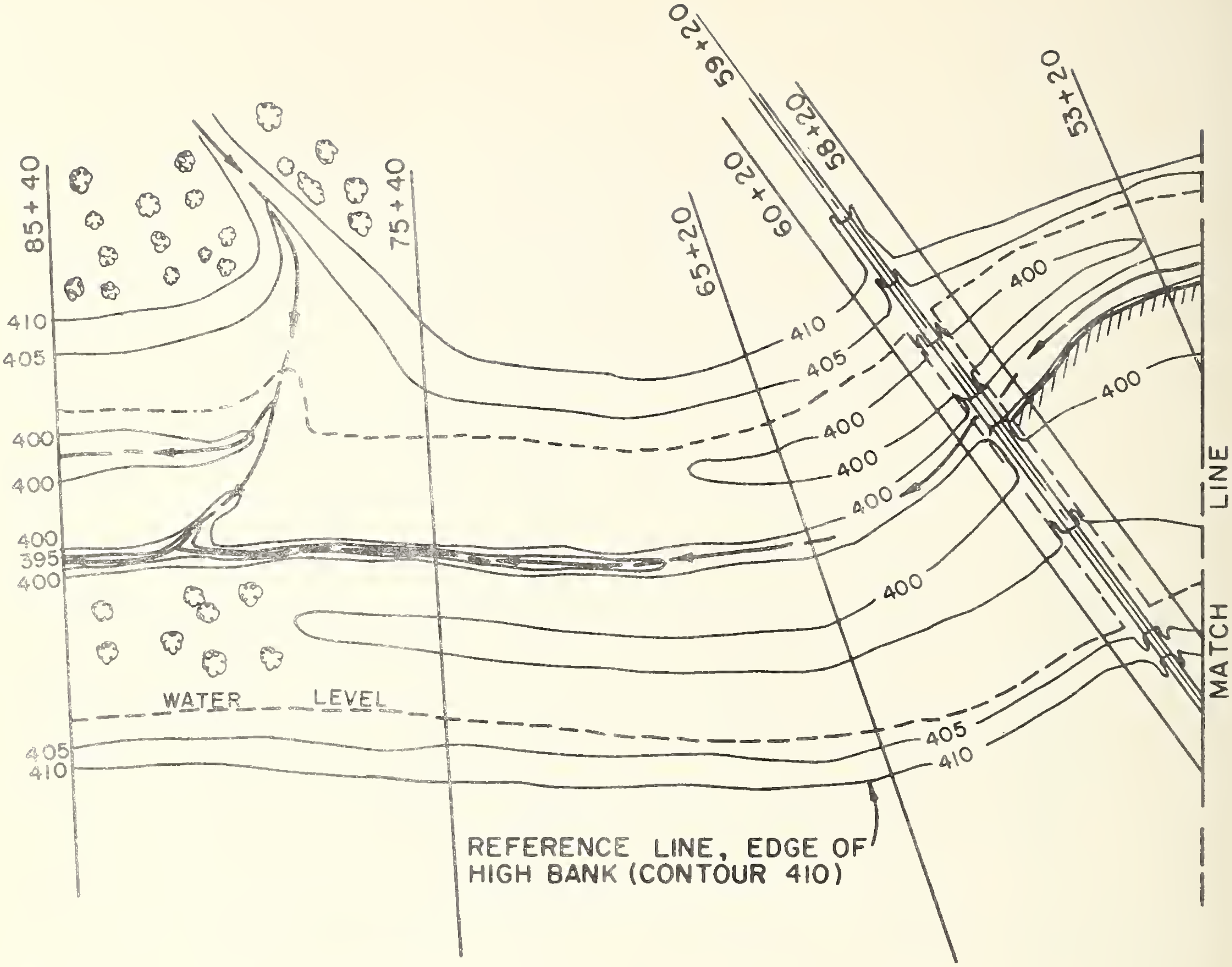
TABLE 1

STREAM DISCHARGE RECORD AT GAGE[1]

Discharge Record		Discharge Array and Plotting Position[2]			
Year	Max. Ann. Daily CFS	Year	n	Discharge CFS	$100\frac{(2n-1)}{(2y)}$
1927	1100	1938	1	1420	1.7
1928	475	1927	2	1100	5.0
1929	550	1939	3	940	8.3
1930	270	1956	4	850	11.7
1931	365	1946	5	800	15.0
1932	350	1951	6	720	18.3
1933	480	1952	7	680	21.7
1934	510	1953	8	610	25.0
1935	430	1945	9	610	28.3
1936	170	1950	10	580	31.7
1937	410	1929	11	550	35.0
1938	1420	1949	12	520	38.3
1939	940	1934	13	510	41.7
1940	330	1933	14	480	45.0
1941	300	1928	15	475	48.3
1942	450	1942	16	450	51.7
1943	290	1935	17	430	55.0
1944	400	1954	18	420	58.3
1945	610	1937	19	410	61.7
1946	800	1944	20	400	65.0
1947	240	1931	21	365	68.3
1948	190	1932	22	350	71.7
1949	520	1940	23	330	75.0
1950	580	1941	24	300	78.3
1951	720	1943	25	290	81.7
1952	680	1930	26	270	85.0
1953	610	1947	27	240	88.3
1954	420	1955	28	235	91.7
1955	235	1948	29	190	95.0
1956	850	1936	30	170	98.3

[1]Gage 1.1 mile upstream from bridge - watershed area 34 sq. mi.

[2]Record organized for Hazen Formula $100\frac{(2n-1)}{(2y)}$ (See Table NEH, Sec. 4, 3.18-2)



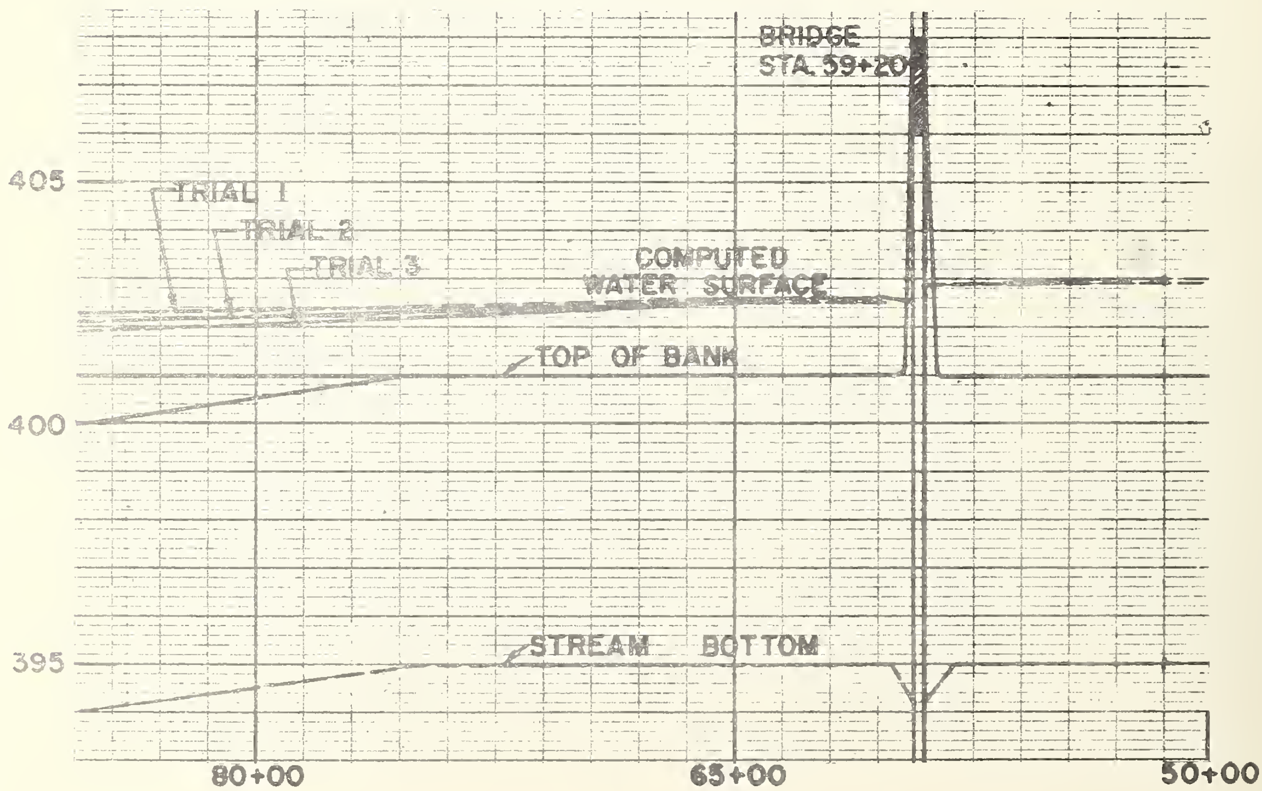
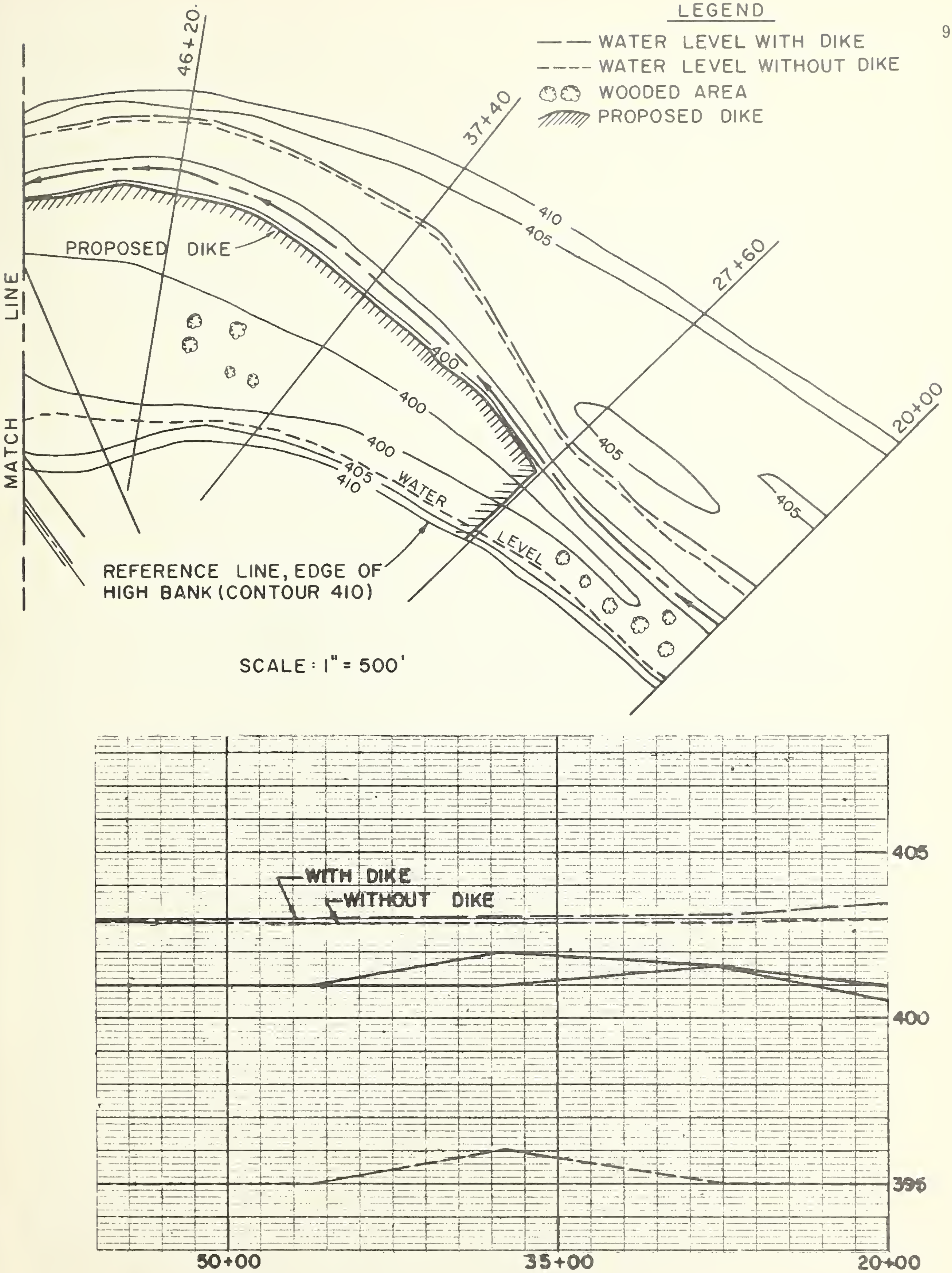


FIGURE 3 — PLAN – PROFILE OF



STREAM CHANNEL AND FLOODWAY

1. Required height of levee.
2. Increased flood stage and extent of flooding on opposite flood plain caused by the levee.

Reconnaissance and Survey.

Surveys must be carried several thousand feet below the bridge to establish control sections for computing the water stage at the starting point of the project. Valley cross sections are located along the channel by direction and stationing and serve as control sections at points of substantial changes in:

1. Drainage Area (entrance of side streams or increments of enlargement of drainage area along stream).
2. Direction of course of stream or floodways.
3. Channel or floodway grade.
4. Depth, width, shape of channel or floodways.
5. Surface roughness determined individually for channel and separate floodways caused by differences in woodland cover, crops, etc.
6. Obstructions or constrictions, as bridge and road embankment.

Supplemental survey data includes measurement of drainage areas; location of supplemental elevations of low or high points in the channel or flood plain between cross sections otherwise having uniform hydraulic properties; and dimensions and elevations of obstructions, such as the bridge, etc.

Plotting of Survey Data.

Plan, profile and cross sections are plotted with control sections located as shown on Figures 3 and 4. In addition, one-foot contours are plotted on the plan from points on the cross sections and interpolations between such sections and the location of intermediate spot elevations. This is done to establish approximate flow paths of out-of-bank flow.

Required Discharge Coefficient.

Discharge for 25-year frequency is determined from a 30-year period of U.S.G.S. gage records. Annual maximum daily discharges are listed in descending order of magnitude and chance occurrence determined by Hazen Formula (See NEH, Section 4, 3.18-2 and Table 1). Values are plotted on log-log probability paper and probability curve established as shown on Figure 5, from which discharge occurring once in 25 years is determined to be 1125 cfs. This discharge divided by the watershed area at the gage (34 sq. miles) gives a coefficient of 33.1 cfs per sq. mile. The coefficient is not adjusted for watershed size due to the small difference in watershed areas at the gage and the downstream site. Therefore, the total required discharge = 37.1 x 33.1 = 1228 cfs.

Hydraulic Data Sheet.

A spread sheet is prepared for orderly tabulation of hydraulic data as computations are completed. Column Headings as shown in Table 2 are self-explanatory and are listed, insofar as possible, across the sheet as hydraulic units are needed or computed.

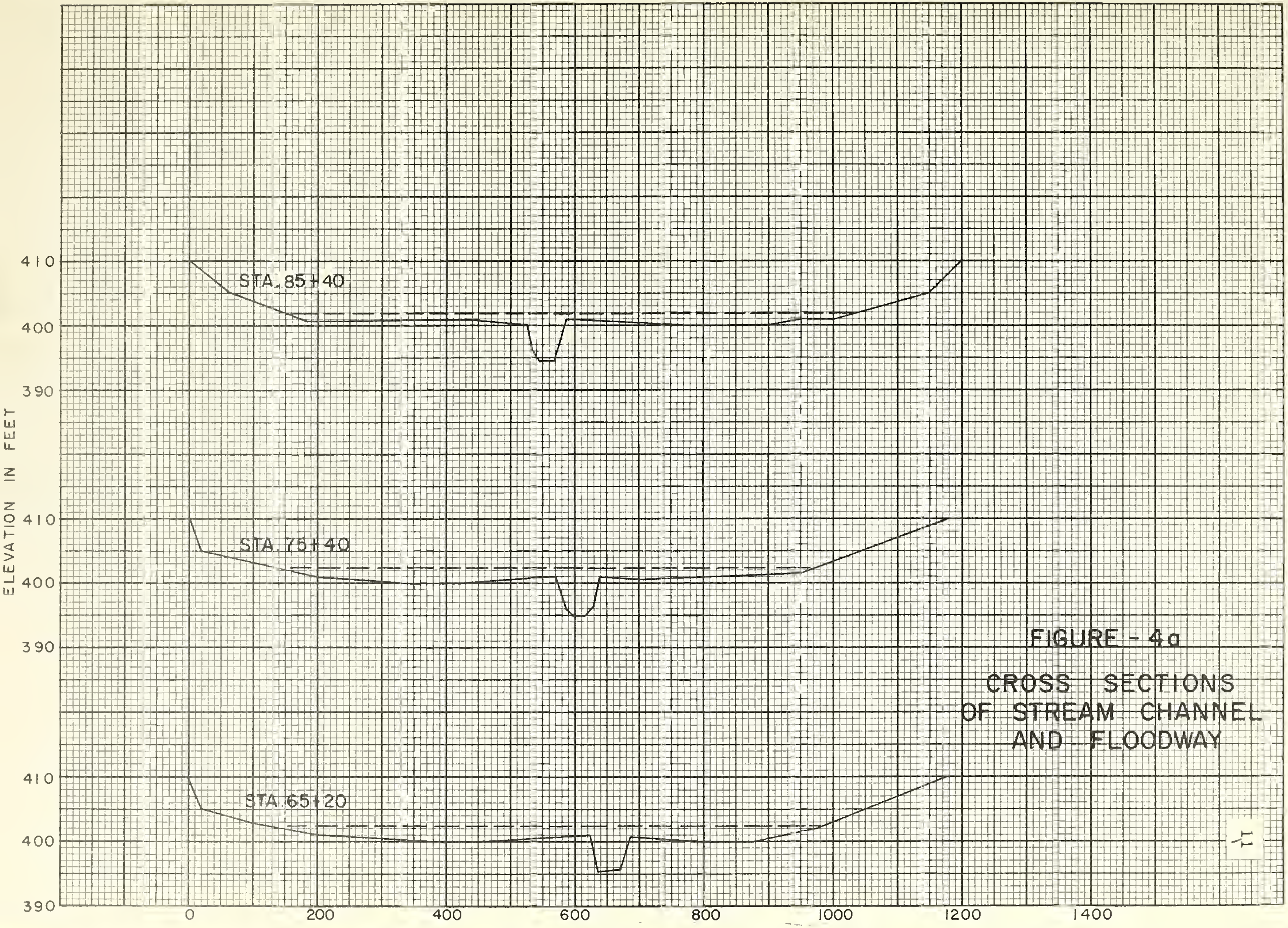


FIGURE - 4a

CROSS SECTIONS OF STREAM CHANNEL AND FLOODWAY

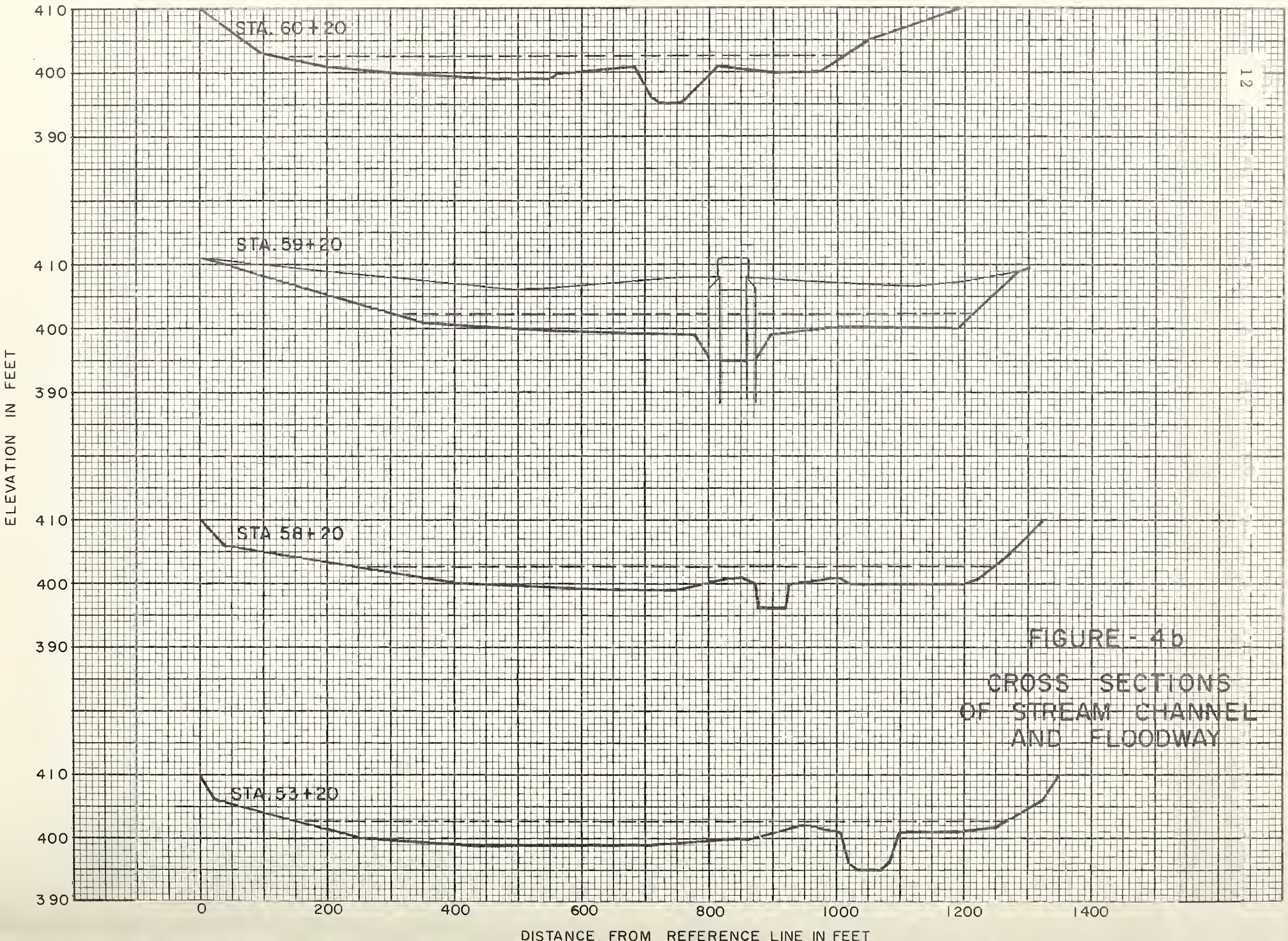


FIGURE - 4b

CROSS SECTIONS OF STREAM CHANNEL AND FLOODWAY

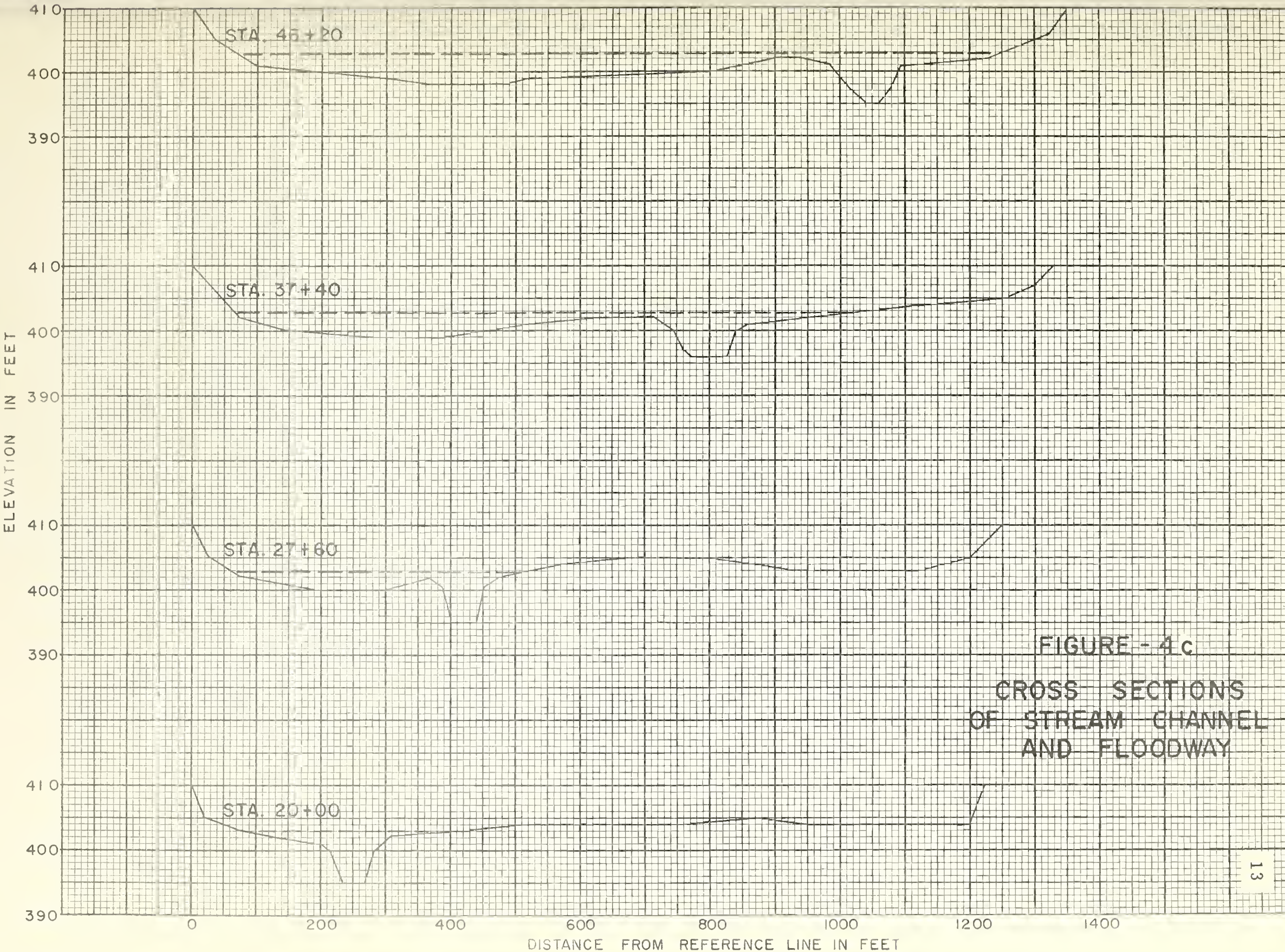


FIGURE - 4c

CROSS SECTIONS OF STREAM CHANNEL AND FLOODWAY

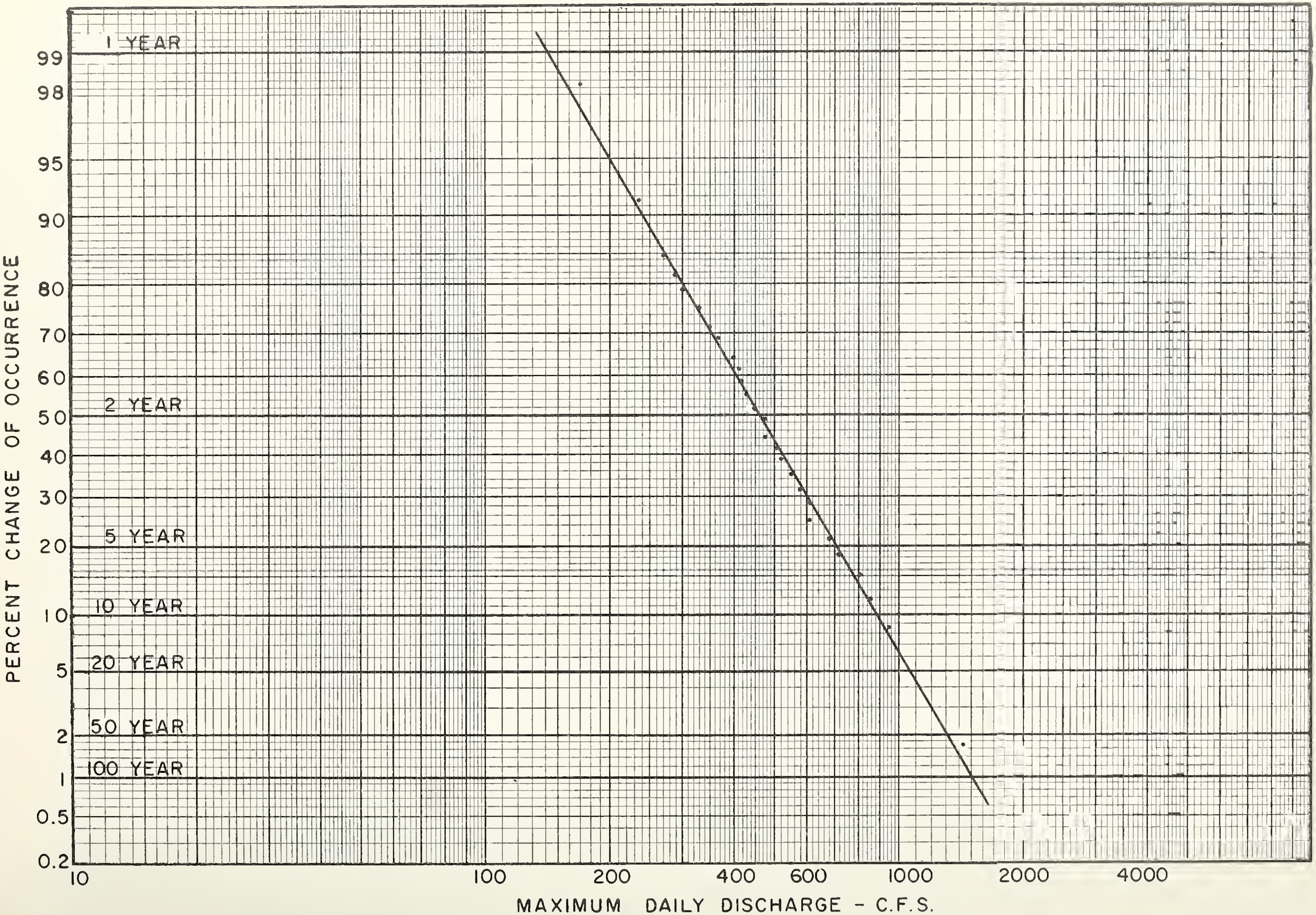


FIGURE 5 - ANNUAL MAXIMUM DAILY STREAM DISCHARGE FREQUENCY AT GAGE

Additional column headings, Nos. 8 through 13, were included for possible use in showing construction dimensions where channel alterations might have been desirable. Each line represents the complete data in establishing water elevations of a single reach.

Stage of Initial Point in Water Surface Profile.

The condition of retarded flow required starting stage to be at the downstream end. Lacking a known or measured stage for the required discharge, several trial profiles are started well downstream from the problem reaches at and above the bridge. Slightly different stages are used, as shown in Table 2, for the several trials. The point of convergence of such trial profiles is the actual stage for the given flow and hydraulic conditions of the channel and floodway through which it moves. Since decreasing stage indicates near convergence of three trials immediately below the bridge and less than .02 ft. difference on the third trial, elevation 402.53 is determined to be the tail water elevation at the bridge.

Procedure for Computing Water Surface Profile Through Unconstricted Reach Below Bridge.

All reaches are considered as having uniform or gradually changing nonuniform flow. Using the first reach of the first trial as an example: (See Line A, Table 2).

1. Insert the stationing of downstream and upstream ends of reach in Columns 1 and 2, respectively.
2. Determine length of reach for channel section by subtracting Column 2 from Column 1 and insert value in Column 4. Since the cross sections indicate parallel flow in the overflow sections, the same reach lengths are inserted in Columns 3 and 5. However, it should be noted that if direction of overflow is not parallel to the channel, as in Line C, different lengths will be obtained and should be accounted for when differences in flow conditions as caused by a change in "n" value require separate computation of parts of the cross section.
3. Assume a starting elevation for the downstream end of the reach and record this elevation in Column 6. Some judgment must be exercised in selecting the first trial elevation to avoid computing an excessive number of trial runs. In this case the elevation was estimated to be slightly above average bank elevation with a water slope that would be flatter than general ground slope. On all subsequent reaches the downstream elevation is already determined, it being the computed upstream elevation of the previous reach.
4. Draw the water surface elevation on the corresponding cross section and stationing in Column 1. (See Figure 4). Determine the area and wetted perimeters separately for channel and both floodplains, for reasons stated above. Cross section areas and wetted perimeters are most readily determined by measurements from cross sections plotted to the same horizontal and vertical scale. Record results in Columns 14 through 19. These areas and perimeters will be considered as applying to

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
	REACH STATIONING		REACH LENGTH			WATER SURFACE ELEVATION		BOTTOM ELEVATION		CHANNEL DIMENSIONS				WETTED CROSS SECTION AREA			WETTED PERIMETER		
	DOWN-STREAM LIMIT	UP-STREAM LIMIT	LEFT FLOOD-WAY (FT)	CHAN-NEL (FT)	RIGHT FLOOD-WAY	DOWN-STREAM LIMIT	UP-STREAM LIMIT	DOWN-STREAM LIMIT	UP-STREAM LIMIT	BOTTOM SLOPE	SIDE SLOPES	BOTTOM DEPTH (FT)	WATER DEPTH (FT)	LEFT FLOOD-WAY $(FT)^2$	CHAN-NEL $(FT)^2$	RIGHT FLOOD-WAY $(FT)^2$	LEFT FLOOD-WAY (FT)	CHAN-NEL (FT)	RIGHT FLOOD-WAY (FT)
	25-YEAR FREQUENCY FLOW OVER CHANNEL AND BOTH BANKS																		
A	85+40	75+40	1000	1000	1000	402.25	402.48	—	—	—	—	—	—	425	380	730	375	61	445
B	75+40	65+20	1020	1020	1020	402.48	402.58	—	—	—	—	—	—	825	400	418	420	75	312
C	65+20	60+20	600	500	440	402.58	402.62	—	—	—	—	—	—	946	355	604	497	65	317
D	60+20	59+20	100	100	100	402.62	402.63	—	—	—	—	—	—	1410	425	575	580	68	280
E	85+40	75+40	1000	1000	1000	402.07	402.36	—	—	—	—	—	—	331	366	629	365	60	436
F	75+40	65+20	1020	1020	1020	402.36	402.48	—	—	—	—	—	—	775	392	381	419	75	308
G	65+20	60+20	600	500	440	402.48	402.53	—	—	—	—	—	—	897	346	572	492	65	315
H	60+20	59+20	100	100	100	402.53	402.54	—	—	—	—	—	—	1352	419	547	580	68	280
I	85+40	75+40	1000	1000	1000	401.94	402.34	—	—	—	—	—	—	250	351	515	350	59	429
J	75+40	65+20	1020	1020	1020	402.34	402.46	—	—	—	—	—	—	766	390	375	418	75	304
K	65+20	60+20	600	500	440	402.46	402.52	—	—	—	—	—	—	887	345	566	492	65	315
L	60+20	59+35	85	85	85	402.52	402.53	—	—	—	—	—	—	1346	418	544	580	68	280
M	59+35	59+05	—	30	—	402.53	402.83	—	—	—	—	—	—	—	332	—	—	56	—
N	59+05	53+20	660	585	585	402.83	402.84	—	—	—	—	—	—	1719	483	900	611	85	320
O	53+20	46+20	450	700	770	402.84	402.85	—	—	—	—	—	—	2497	645	241	855	100	170
P	46+20	37+40	575	880	920	402.85	402.86	—	—	—	—	—	—	2904	568	198	825	95	150
Q	37+40	27+60	960	980	990	402.86	402.89	—	—	—	—	—	—	1619	780	244	715	115	210
R	27+60	20+00	800	760	760	402.89	403.00	—	—	—	—	—	—	671	489	47	310	86	60
S	20+00	10+00	1000	1000	1000	403.00	403.16	—	—	—	—	—	—	169	517	119	140	72	115
	25-YEAR FREQUENCY FLOOD FLOW-EXCLUDING LEFT BANK ABOVE BRIDGE																		
T	59+05	53+20	—	585	585	402.83	402.88	—	—	—	—	—	—	—	483	900	—	85	320
U	53+20	46+20	—	700	770	402.88	402.96	—	—	—	—	—	—	—	649	248	—	100	170
V	46+20	37+40	—	880	920	402.96	403.10	—	—	—	—	—	—	—	578	215	—	95	150
W	37+40	27+60	—	980	990	403.10	403.18	—	—	—	—	—	—	—	806	309	—	115	272
X	27+60	20+00	—	760	760	403.18	403.38	—	—	—	—	—	—	—	514	70	—	87	70
Y	20+00	10+00	—	1000	1000	403.38	403.53	—	—	—	—	—	—	—	544	172	—	73	140

TABLE 2 - HYDRAULIC DATA FOR

20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39
HYDRAULIC RADIUS			ROUGHNESS COEFFICIENT			REQUIRED DISCHARGE			HYDRAULIC SLOPE			VELOCITY			AVAILABLE DISCHARGE				REMARKS
LEFT FLOOD-WAY	CHAN-NEL	RIGHT FLOOD-WAY	LEFT FLOOD-WAY	CHAN-NEL	RIGHT FLOOD-WAY	AREA (SQ.MI.)	COEF. (CFS/SQ MI)	Q_R (C.F.S.)	LEFT FLOOD-WAY	CHAN-NEL	RIGHT FLOOD-WAY	LEFT FLOOD-WAY (FS)	CHAN-NEL (FS)	RIGHT FLOOD-WAY (FS)	LEFT FLOOD-WAY (CFS)	CHAN-NEL (CFS)	RIGHT FLOOD-WAY (CFS)	TOTAL (CFS)	
25-YEAR FREQUENCY FLOW OVER CHANNEL AND BOTH BANKS																			
1.13	6.28	1.65	.04	.04	.08	37.1	33.1	1228	00023	00023	00023	.61	1.92	.34	259	730	248	1237	1ST TRIAL
1.97	5.34	1.34	.04	.035	.04	36.1	33.1	1195	00010	00010	00010	.58	1.30	.45	478	520	188	1186	
1.90	5.46	1.90	.04	.035	.045	36.0	33.1	1192	00007	00008	00009	.49	1.24	.50	463	440	302	1206	
2.53	6.25	2.05	.045	.040	.045	36.0	33.1	1192	00005	00005	00005	.43	.89	.38	606	378	219	1203	
.90	6.10	1.45	.04	.04	.08	37.1	33.1	1228	00029	00029	00029	.59	2.11	.39	195	772	246	1213	2ND TRIAL
1.84	5.23	1.24	.04	.035	.04	36.1	33.1	1195	00012	00012	00012	.61	1.39	.47	473	545	179	1179	
1.85	5.33	1.82	.04	.035	.045	36.0	33.1	1192	00008	00010	00011	.50	1.30	.51	448	439	291	1178	
2.30	6.17	1.95	.045	.04	.045	36.0	33.1	1192	00005	00005	00005	.41	.89	.37	558	373	206	1137	
.71	5.95	1.22	.04	.04	.08	37.1	33.1	1228	00040	00040	00040	.59	2.44	.43	148	856	222	1226	3RD TRIAL
1.83	5.20	1.23	.04	.035	.04	36.1	33.1	1195	00012	00012	00012	.60	1.39	.46	460	543	172	1175	
1.80	5.31	1.80	.04	.035	.045	36.0	33.1	1192	00008	00011	00011	.49	1.29	.51	435	445	259	1169	
2.32	6.15	1.94	.045	.04	.045	36.0	33.1	1192	00006	00006	00006	.45	.96	.39	605	401	212	1218	
—	5.75	—	—	—	—	36.0	33.1	1192	—	—	—	—	—	—	—	—	—	1192	HEADING THRU BRIDGE STA. 59+20
2.81	5.68	2.82	.040	.035	.040	36.0	33.1	1192	00002	00002	00002	.33	.60	.33	567	290	297	1154	SECTION AT 58+20
2.89	6.45	1.42	.055	.035	.040	35.9	33.1	1188	00003	00002	00002	.30	.66	.21	749	426	51	1226	
3.52	5.98	1.34	.050	.035	.040	35.8	33.1	1185	00002	00001	00001	.31	.44	.14	900	250	28	1178	
2.26	6.87	1.16	.050	.035	.040	35.7	33.1	1182	00003	00003	00003	.28	.84	.23	453	655	49	1157	37 FT. ON LEFT BANK EXCLUDED
2.16	5.69	.78	.060	.035	.045	35.6	33.1	1178	00014	00015	00015	.49	1.66	.35	329	810	16	1155	
1.21	7.18	1.04	.060	.035	.045	35.5	33.1	1175	00016	00016	00016	.36	2.06	.43	61	1034	51	1146	
25-YEAR FREQUENCY FLOOD FLOW - EXCLUDING LEFT BANK ABOVE BRIDGE																			
—	5.68	2.82	—	.035	.040	36.0	33.1	1192	—	00008	00008	—	1.19	.65	—	575	585	1160	
—	6.49	1.46	—	.035	.040	35.9	33.1	1188	—	00012	00011	—	1.61	.50	—	1045	124	1169	
—	6.08	1.47	—	.035	.040	35.8	33.1	1185	—	00016	00015	—	1.78	.59	—	1029	127	1156	
—	7.00	1.14	—	.035	.040	35.7	33.1	1182	—	00008	00008	—	1.37	.36	—	1105	111	1216	
—	5.92	1.00	—	.035	.040	35.6	33.1	1178	—	00026	00026	—	2.23	.60	—	1146	42	1188	
—	7.45	1.23	—	.035	.040	35.5	33.1	1175	—	00015	00015	—	1.98	.52		1077	89	1166	

STREAM CHANNEL AND FLOODWAY

the entire reach length except as follows: With "Q" the same for successive reaches, velocities between succeeding reaches when greater than 2-1/2 to 3 feet per second should not differ by more than 20 percent. In such cases a mean section average of end areas of the two reaches can be computed and used as a transition section.

5. Record the roughness coefficients for parts of the cross section in Columns 23 to 25 from field notes. The required discharge (Column 28) is next computed from the coefficient (Column 27) and drainage area (Column 26).
6. Estimate a trial hydraulic slope for the main channel and insert in Column 30. The difference in water elevation is then computed for the reach length in Column 4. If reach lengths for floodway sections, Column 3 or 5, differ from Column 4, then slopes will be determined by dividing this difference in elevation by Columns 3 or 5 and inserting the respective slopes in Columns 29 or 31. The difference in elevation added to Column 6 will give the upstream elevation (Column 7). If total discharge for Columns 35, 36 and 37 is not equal to the required discharge, Column 28, as computed from velocities (Columns 32, 33 and 34) and corresponding hydraulic slopes in Columns 29, 30 and 31, a new slope must be selected in Column 30 and the process repeated. In reaches where values of "n" and reach length are the same so that computation can be considered as one unit, it is much more simple to determine the velocity by first dividing Column 28 by the total of Columns 14, 15 and 16 and then determine the water slope common to Columns 29, 30 and 31.

<u>Procedure for Computing Water Surface Profile Through Constricted Reach at Bridge</u>. (Line M, Table 2).

With head and tail water elevations as the respective top and bottom of reaches, compute the head loss through the bridge. This can be done as a special orifice using the Yarnell Culvert Flow Formula (Kings Handbook, Fourth Edition, 3-23) for square cornered entrance corrected for 45° wing walls (Kings Handbook, Fourth Edition, 3-24). Although suppression does not occur as in a full flowing culvert coefficient, the error is small.

Length of Waterway = 35 ft.

Area of Waterway $= \dfrac{7.54 + 8.54}{2} \times 40 = 322$ sq. ft.

Wetted Perimeter $= 7.54 + 40 + 8.54 = 56$ ft.

Hydraulic Radius $= 322/56 = 5.75$ ft.

$$C = \left(1 + 0.4r^{0.3} + \frac{.0045L}{r^{1.25}}\right)^{-1/2} = 0.787$$

Add 10% allowance for 45° wing walls

$$C = 0.767 \times 1.1 = 0.84$$

$$Q = CA\sqrt{2g\ (h + h^1)} \quad \text{or} \quad \sqrt{2g\ (h + h^1)} = \frac{Q}{CA}$$

(Since velocity of approach would be less than that for the downstream reach or less than 0.5 ft. per second, h^1 is negligible and therefore omitted.)

$$\text{Then } \sqrt{2gh} = \frac{1192}{.84 \times 322} = 4.41 \text{ ft. per second}$$

$h = .30$ ft. value to be added to Column 6

(El. 402.53) to give the upstream water elevation, Column 7, at upstream end of bridge (El. 402.83).

Procedure for Computing Water Surface Profile Above Bridge.

Compute the water profile elevations above the bridge, Lines N through S (Table 2), for the 25-year discharge without the dike. Next, compute water surface profile, Lines T through Y, for the same discharge with dike in place by omitting the left bank from the cross section. Plot water elevations on plan and profile sheet No. 3.

Results and Conclusions.

The plotted water surface profile shows that a levee elevation of 404, plus the required freeboard, is ample for protection against 25-year discharges and that an increased stage after diking is small and will cause little additional flooding to agricultural lands on the opposite streambank.

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